Wastewater Flow Monitoring Services

For

K C Design
Architecture - Interiors

San Gabriel, CA

July 13, 2015 through July 21, 2015

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US3
Leaders in Sewer Flow Monitoring Services

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Tab 1: Overview & Summary of Site

1.1 Overview

The City of San Gabriel owns and operates over 72 miles of sewer and 1,300 manholes. The main sewer system is located below public streets and, in some cases, within easements that run along the back of private properties. Untreated wastewater is collected in the system and delivered to the trunk line system of the County Sanitation Districts (CSD). The CSD then treats and discharges the wastewater. The City's sewer collection system is operated under a state-issued permit and must comply with Federal and State water quality laws.

The City is responsible for properly managing, operating, and maintaining all portions of their wastewater collection system, as well as providing adequate capacity to carry peak capacity flows, protection of surface and ground water supplies, protection of public health by minimizing the frequency of Sewer System Overflows (SSOs), mitigating the impacts that are associated with all SSOs that may occur, and complying with all applicable regulatory notification and reporting requirements. Therefore, the City of San Gabriel prepared a comprehensive study of the public sewer system in 2009. The study included an inventory of the pipes and manholes that make up the system, an evaluation of the sewer flows and capacity of the system, and an assessment of the condition of the system.

1.2 Project Description

As a condition of development, sewer flow monitoring is required as a means of continually updating the City database with the most current sewer capacitance information. Utility Systems Science & Software, Inc. (US³) was contracted by KC Design to install flow monitoring equipment at a specifically selected sewer manhole downstream from the proposed development at 237 E. Las Tunas Dr. This sewer system assessment was developed for the current conditions to determine existing capacitance. The selected manhole (MH 621) was located at the intersection of E. Las Tunas Dr and St. Francis St. The manhole and project locations as well as the sewer lines are shown on maps in Section 1.3.

US³ performed an initial site inspection to assess traffic control requirements and evaluate the site hydraulics at MH 621. Note: Each wastewater collection system is different, and some of the differences that affect flow monitoring include: slope of sewer line; age, condition and material of sewer line; number and type of connections; and sewer design, specifically the proximity of baffles, drop structures, pumps, siphons, and weirs. An elevated lateral was discharging flow from the west into MH 621, which created hydraulics that were not conducive to flow monitoring; therefore, the next downstream manhole (MH
555R) was inspected. The hydraulics within this manhole were found to be acceptable; therefore, it was determined that MH 555R should provide an accurate representation of the flow patterns within the upstream sewer line. The City approved of the installation site.

Prior to installation of flow monitoring equipment, Downstream Services was contracted to clean the sewer lines adjacent to the proposed project and perform visual inspections using CCTV technology. Therefore, the lines were cleaned and inspected from MH 133 east to MH 621, from MH 193 south to MH 194, from MH 194 west to MH 621, and from MH 621 south to MH 555R. Installation of flow monitoring equipment at MH 555R followed the cleaning and inspection process on 7/13/2015.

The manhole was equipped with Hach Marsh-McBirney Flo-Dar® AV Sensor and Logger. With the installation and removal of the equipment, the manhole was opened and confined space entry was limited to crossing the plane with hands and arms, but not actually entering the manhole. Even though US³ personnel did not enter the manhole during installation, they were fully trained and certified in Confined Space Entry and CPR, and in accordance with safety standards, gas readings were taken and recorded throughout the duration of the installation and removal process.

In summary, US³ performed the following services as part of this project:
- Sewer Flow Monitoring
  - Detailed preliminary investigation
    - Assessment of traffic control requirements
    - Inspection and evaluation of the site
  - Installation of a Marsh-McBirney Flo-Dar® AV Sensor and Logger
    - Validation of hydraulic suitability
    - Calibration of MM Meter
    - Record of detailed site data
  - Flow monitoring between 7/13/2015 and 7/21/2015
  - Removal of flow monitoring equipment
- Validation of flow monitoring data
- Evaluation of the sanitary sewer line at the selected site
- Development of a report to discuss the flow monitoring data, including system statistics and graphs and the CCTV inspection results
1.3 Site Maps

Figures (above & below): The temporary wastewater flow monitoring location is at the intersection of E. Las Tunas Dr and St. Francis St. MH 555R is located at latitude 34.102994 and longitude -118.097072.
Figure above: Sewer Map with cleaning, inspection and monitoring locations identified from City of San Gabriel Engineer.
1.4 Summary of Site

Figures below: The photos show MH 555R at the intersection of E. Las Tunas Dr and St. Francis St, which was a commercial/residential area with light traffic. Therefore, appropriate traffic control was required by the field team (cones and signs).

Figure above (MH 621) & right (MH 555R): Both manholes had elevated laterals entering from the west, but the lateral in MH 555R appeared abandoned while the lateral in MH 621 was discharging into the center of the downstream flow. Since this was noticeably disrupting the hydraulics, MH 555R was used for monitoring.
Figure above: Four sewer lines entered MH 555R (one from the east, one from the north and two from the west) and one sewer line discharged to the south. Gas levels were good. The flow monitoring equipment was set to monitor the upstream 8 inch pipe, which entered the manhole from the north. Follow up on this installation confirmed equipment calibration.

1.5 Statistical Analysis

Figure below: Site graph of flow, level & velocity and scatter plot of level in relation to velocity from US³ Flow Monitoring Website (http://flodar.uscubed.com)
The capacity evaluation method involves the following process:

1. Collect 15-minute interval depth and velocity data points in the pipe over the entire monitoring period and determine the depth vs. velocity relationship or pipe curve based on Manning’s equation.

2. Determine statistically the minimum, maximum and average depths and flow rates in the pipe during the monitoring period.

3. Calculate the maximum theoretical unobstructed flow rate for the pipe.

4. Determine remaining capacity at peak flow conditions in the pipe, using the ratios of actual daily flow depths to pipe diameter.

The scatter plot above shows the depth vs. velocity data points for the monitored pipe. The pattern of velocity and depth data points for a sewer operating in free-flow conditions over an extended period of time should conform to the depth-velocity relationship of the Manning Equation, which is the basis for evaluating flow meter data. This equation (shown below) defines a commonly used theoretical relationship between depth and velocity in pipes operating under free flow conditions:

\[
v = \frac{1.486}{n} \times R^{2/3} \times S^{1/2}
\]

Where,

\[
\begin{align*}
  v & = \text{velocity (fps)} \\
  n & = \text{pipe roughness, defined as the Manning’s Roughness Coefficient} \\
  R & = \text{hydraulic radius,}
      \text{defined as wetted area (ft}^2\text{) divided by wetted perimeter (ft)} \\
  S & = \text{hydraulic slope}
\end{align*}
\]

**Figure at right:** This theoretical pipe curve was generated for an 8 inch pipe with a slope of 0.25%.
The flow monitoring data includes flows from 0.18 ft through 0.40 ft in depth with velocities from zero to 1.68 fps. Therefore, the data points in the scatter plot for MH 555R do not resemble the above theoretical hydraulic operating curve, which was based on the site specific characteristics of the monitored pipe.

To summarize the site statistics presented on the following page:

- During the peak flow, 67.9% of the pipe’s capacity was utilized at 128.95 gpm.
- Average flows utilized 32.7% of the pipe’s capacity.
- According to common sewer design, velocity shall not be less than 2 fps a minimum of once per day to provide sufficient scouring action for self-cleaning. The maximum velocity at MH 555R was 1.68 fps, which only occurred for 0.5 hrs on 7/20, while the average was 0.93 fps. Therefore, this site is likely to have blockage due to settlement of normal system solids. Where 2 fps velocity cannot be provided at least once per day, the slope should be at least 0.01 ft/ft.
- According to common sewer design, the depth versus diameter (d/D) ratio for gravity drains of 10 inches in diameter or less should be no greater than 0.50 for the ultimate peak flow condition. The average d/D ratio for the site during this study was 0.41 and the maximum was 0.60. Therefore, this site has no available capacity at peak flow.

The following is a graphical representation of the site statistics from the data analysis:
The pipe capacity was estimated using a Manning’s Circular Pipe Flow Capacity chart, where
n=0.013 (minimum value for vitrified sewer with manholes, inlets, etc., Chow 1959). Actual
velocities and levels were compared to theoretical hydraulic operating curves to determine which
slope most closely correlates with data. The result was a slope of ~0.25% (S=0.0025) at MH 555R.
Therefore, with the pipe completely full, the velocity should be ~1.215 fps, which when input into
the Flow-Ware program produced a maximum capacity of 0.2736 mgd. This is a reasonable
assumption based on the flows observed during this study.

1.6 Proposed Flow Generation
The proposed project at 237 E. Las Tunas Drive is approximately 0.61 acres in size. With
regard to sewerage service, the expected average wastewater flow (Q_{AF}) from the
proposed project is 13 gallons per minute (gpm). In accordance with common sewer
capacity study requirements, the peaking factor used to determine peak flow (Q_{PF}) for
sewer diameters less than 15 inches is 2.5.

<table>
<thead>
<tr>
<th>Q_{AF} (gpm)</th>
<th>Q_{AF} (mgd)</th>
<th>2.5 x Q_{AF} = Q_{PF} (mgd)</th>
<th>Estimated Total Flow under Peak Flow Conditions (mgd)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Proposed Project</td>
<td>13</td>
<td>0.0187</td>
<td>0.0468</td>
</tr>
</tbody>
</table>
1.7 Conclusions

According to the CCTV inspection, between MH 133 and MH 621, there was one deposit of attached ragging and two deposits of attached grease as well as two locations with fractures and three locations with cracks. Between MH 193 and MH 194, there were 18 locations with cracks. Between MH 194 and MH 621, there was one location with fractures and five locations with cracks, and between MH 621 and MH 555, there was also one location with fractures and five locations with cracks. No root intrusion was noted in any of the lines.

The graphs for the site show regular daily flow patterns for the first seven days representative of a retired community with a commercial component. On the weekdays, the flow picked up in the mornings at ~0800 and slowed at night at ~2300. On the weekend, the flow picked up a little later in the day, but on the last two days of the study (Monday 7/20 and Tuesday 7/21), the pattern changed and there were increases in both the level and velocity. According to the nearest weather station (San Gabriel KCASANGA7), which is located ~1400 feet NW of the site on N. Del Mar Av, there was 0.17 inches of rain on 7/18 and 0.02 inches on 7/19. Therefore, this could have been the result of infiltration from the rain event.

The peak flow of 0.1857 mgd occurred at 0830 on 7/20 and the d/D design criteria ratio of 0.5 was exceeded. In addition, there were two other days on which the 0.5 d/D design criteria ratio was exceeded, occurring on 7/14 from 0800 to 1015 and on 7/16 from 1000 to 1030. On 7/19, flows exceeded the 0.5 d/D ratio limit starting at 2345 until 0015 on 7/20, then continuing on 7/20 from 0530 to 1030 and ending with 2130 to 2230. However, throughout the monitoring period, only 5.4% of the data exceeded the 0.5 d/D design criteria ratio, and by less than 4% on all occasions except on 7/20.

The velocities dropped below readable levels from 2.25 hrs to 4 hrs between 0145 and 0600 every day except for 7/19 and 7/21. Studies have shown that Flo-Dar sensors provide accurate velocity readings down to 0.4 fps, but below 0.4 fps, extremely laminar flow makes radar readings much less reliable. Therefore, the cutoff was set at 0.4 fps and any readings below that cutoff were recorded as zero. The consistently low velocities prevented good capacitance within the line and will promote settlement of normal system solids.

Using the maximum velocity of 1.68 fps, MH 555R could convey 0.3784 mgd at full pipe and 0.1895 mgd at half pipe. However, since the velocity only reached 1.68 fps twice on 7/20, once at 1545 with a level of 3.61 inches and once at 1630 with a level of 3.55 inches, it is unlikely that this velocity can be expected during peak flow events. Therefore, using the velocity that accompanied the maximum level (4.82 inches) of 1.31 fps, the site could reasonably convey 0.2950 mgd at full pipe. Using the theoretical hydraulic operating curve
that most closely correlates with the data, the velocity at half pipe should be 1.16 fps, which would convey 0.1309 mgd before the pipe exceeded its d/D design criteria limit of 0.50. Even though velocities occasionally exceed the expected theoretical values at different levels thereby skewing the scatter plot, it is not consistent. Therefore, even though the existing flows are within the conveyance capacity of the pipe, they occasionally exceed the common sewer design criteria with regard to d/D limitations.

Due to the occasional d/D design criteria exceedance during the flow monitoring study, a spreadsheet was created showing the addition of the proposed peak flow generation and the estimated depth that can be expected during ultimate peak flow conditions. As shown in the following table, the average d/D ratio with 0.0468 mgd added to every data point was 0.526 and the maximum was 0.704.